Vermont Flood Plain Management Services

Dam-Break Flood Analysis
Chester Reservoir
Chester, Vermont

March 1994

US Army Corps
of Engineers
New England Division

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13. ABSTRACT *(Maximum 200 words)* The purpose of this study was to determine the downstream hazard potential of Chester Reservoir in Chester, Vermont. Dam-break flood conditions were evaluated for both sunny-day and storm-day failures. Using the hydrological model HEC-1, five storms, the 100 year storm and four fractions(1,3/4,1/2, and 1/4) of the probable maximum precipitation(PMP) were analyzed. The inflow hydrograph which resulted in a maximum water depth of about 2 feet above the dam crest was considered to be the inflow condition for causing storm-day (overtopping) failure. This inflow was determined to be the 1/2 PMF. Two dam-failure floods were analyzed using the National Weather Service DAMBRK Flood Forecasting Model. The analysis covered an area of about 3.2 miles along the downstream channel. This dam is rated Class 2 or a Significant hazard category because of its small size and its potential to cause downstream damage, in terms of either loss of life or economic loss. Introductory information was also developed to aid in the development of an Emergency Action Plan in the event of an impending dam failure.

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DAM-BREAK FLOOD ANALYSIS

CHESTER RESERVOIR
CHESTER, VERMONT

Prepared for

The U.S. Army Corps of Engineers
New England Division

at the request of

State of Vermont
Department of Environmental Conservation
Dam Safety Program

Prepared by

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Contract No.: DACW 33-92-D0003

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EXECUTIVE SUMMARY

The primary purpose of this study is to determine the downstream hazard potential of Chester Reservoir for the Dam Safety Program under the jurisdiction of the State of Vermont, Department of Environmental Conservation. The secondary purpose of the study is to provide introductory information for the dam owner and to develop an Emergency Action Plan (EAP) for impending dam failure.

Dam-break flood conditions are evaluated for both sunny-day and storm-day failures. Sunny-day failure was assumed to occur with minimum inflow to the reservoir. Storm-day failure was assumed to be caused by a significant inflow hydrograph. Five storms, the 100-year storm and four fractions (1, ¾, ½ and ¼) of the probable maximum precipitation (PMP) were analyzed. The hydrological model HEC-1 was employed to develop inflow hydrograph resulting from each of the five storms, and to determine the corresponding outflow hydrograph. The inflow hydrograph which resulted in a maximum water depth of about 2 feet above the dam crest was considered to be the inflow condition for causing storm-day (overtopping) failure. This inflow was determined to be the ¾ probable maximum flood (PMF).

The two dam-failure floods were analyzed using the National Weather Service DAMBRK Flood Forecasting Model. The analysis covered a reach of about 2.6 miles along the downstream channel. Peak stages and flows at various locations along the channel were determined. Maps of inundation caused by the floods are provided.

On the basis of the U.S. Army Corps of Engineers' guidelines for dam safety inspection, Chester Dam is classified as SMALL by its size. On the basis of its potential to cause downstream damage, in terms of either loss of life or economic loss, the dam is rated Class 2 or a SIGNIFICANT hazard category.

Four major components of the EAP are discussed: monitoring, evaluation, preventive action, and warning. The EAP also includes a current listing of officials to contact in the event of an impending dam failure.

A. DAM-BREAK FLOOD ANALYSIS

1. INTRODUCTION

a. Purpose

This report presents the findings of a dam-break flood analysis performed for Chester Reservoir located in Chester, Vermont. The dam is owned, operated and maintained by the Town of Chester. The purpose of this investigation was to evaluate the effect of a hypothetical dam-break flood in the downstream valley and to determine hazard classification of the dam. The investigation was not performed because of any known likelihood of a breach of Chester Dam.

The report provides a description of pertinent features of the watershed, reservoir, and dam. Procedure of the dam-break analysis, conditions for dam-break and resulting flooding effect in downstream areas are discussed in detail. Important results include: downstream hydrographs; peak flows, peak stages and their timing at all surveyed river cross-sections; inundation maps for the river reach under study. The report also provides a current listing of local and state officials to contact in the event of a dam failure.

b. Authority

The U.S. Army Corps of Engineers, New England Division authorized Hydraulic and Water Resources Engineers, Inc. of Waltham, Massachusetts to conduct this dam-break study at the request of the Vermont Department of Environmental Conservation. The study was funded through the Corps of Engineers Section 206 Flood Plain Management Services (FPMS) Program.

c. Downstream Hazard Classification

Dams are classified according to their potential to cause loss of life and property damage in the area downstream of the dam if it were to fail. The hazard classification does not refer to the condition of the dam.

The classification system used in this study has been adopted by the U.S. Army Corps of Engineers and is used by the Vermont Department of Environmental Conservation to determine inspection frequency and spillway adequacy for dams under its jurisdiction. The categories and criteria for the hazard classification of dams, as reported in "Recommended Guidelines For Safety Inspection of Dams", Department of the Army, Sept. 1979 (Ref. 1), are listed in the following table.

DAM HAZARD CLASSIFICATION

Class	Potential Hazard Category	Loss of Life (Extent of Development)	Economic Loss (Extent of Development)
3	Low	None expected (No permanent structures for human habitation)	Minimal (Undeveloped to occasional structures or agriculture)
2	Significant	Few (No urban developments and no more than a small number of inhabitable structures)	Appreciable (Notable agriculture, industry or structures)
1	High	More than a few	Excessive (Extensive community, industry or agriculture)

2. PROJECT DESCRIPTION

a. General

Chester Reservoir, formed by an earth fill dam, is located in south Vermont in the town of Chester (Fig. 1). The dam was constructed in 1910 for water supply but currently does not serve that purpose. The length of the dam is approximately 220 feet, height is 20 feet and crest width is about 17 feet. The dam has a principal spillway (7 feet wide and 2.4 feet deep) and an emergency spillway (8 feet wide and 1.5 feet deep), both being concrete chutes (Fig. 2). The dam has been upgraded twice: leveling of embankment in 1950 and replacing the original valve house with a 6-inch drain pipe in 1971.

Chester Reservoir has a drainage area of 0.6 square miles. At normal pool elevation, the reservoir surface area is about 4 acres. The two spillways, as reported in the inventory prepared by the U.S. Army Corps of Engineers, have a combined maximum design discharge of 155 cubic feet per second (cfs) with the pool at top of dam. The discharge channel from the dam flows into a small brook that merges with Middle Branch Williams River about 1 mile downstream from the dam. This study covers the 1-mile stream and 1.6-mile reach of Middle Branch Williams River, a total length of 2.6-miles as shown in Fig. 1.

b. Community Description

The neighboring community is the Town of Chester. Within the 1-mile reach of the stream from Chester Reservoir, there are scattered residential houses on both banks. Most of the town is on the left bank of Middle Branch Williams River. Route 11 follows the river and runs through the town. The town center is about 2.6-miles downstream from Chester Reservoir.

c. Downstream Conditions

The stream initiating from the reservoir is a typical mountainous stream, narrow and steep. Channel bottom elevation is about 1045 feet above National Geographic Vertical Datum (NGVD, hereafter elevation is referred to NGVD) at the dam, and drops to about 707 feet at Route 11 (0.91 miles from the dam). Average channel slope is 370 feet/mile. At some locations, the slope is as high as 500 feet/mile. The slope of Middle Branch Williams River is about 60 feet/mile on the average within a 1-mile reach downstream from the river's confluence with the reservoir stream. Flood plains of the river are expected to provide some storage during a flood.

The reservoir stream passes through 5 culverts along its course from the dam to its confluence with Middle Branch Williams River. The first three culverts are rectangular with widths of 4 feet, and heights varying from 4 to 8 feet. The last two culverts are formed by metal pipes with a diameter of 3 feet. There is a bridge across Middle Branch Williams River about 1.6-miles downstream from the dam, leading to several residential houses on the right bank. At the end of the study reach, about 2.6 miles downstream from the dam, is a foot bridge leading to a school on the right bank.

3. DAM DESCRIPTION

a. Identification

Chester Dam is identified by the State of Vermont as 48-1. The national inventory prepared by the U.S. Army Corps of Engineers identifies this dam as VT00300.

b. Physical Characteristics

Type:	Earth fill
Length:	220 ft.
Height:	20 ft.
Top Width:	17 ft.
Side Slope:	Upstream face, 3:1 (Horizontal to Vertical). Downstream face, 2:1 (Horizontal to Vertical).

c. Spillways

Principal Spillway:

Type: 7-foot wide weir
Maximum Hydraulic Capacity: 68 cfs. (computed with a discharge coefficient of 2.6 and water surface at top of dam)

Emergency Spillway:

Type: 8-foot wide weir
Maximum Hydraulic Capacity: 37 cfs. (computed with a discharge coefficient of 2.6 and water surface at top of dam)

d. Impoundment Behind Dam

Surface Area:

At principal spillway crest	4 acres
At top of dam	5 acres

Height of Dam: 20 feet

Storage Volume (from Vermont State Dam Inventory):

At principal spillway crest	31 ac-feet
At top of dam	46 ac-feet

e. Dam Site Elevations (referred to NGVD)

Top of dam	1065.0 ft

Principal spillway	1062.6 ft
Emergency spillway	1063.5 ft
Low drain pipe outlet	1045.0 ft
Streambed at downstream toe of dam	1045.0 ft

f. Watershed Area

Size: 0.6 square miles
Type: Primarily woodland and undulating terrain

4. METHOD OF ANALYSIS

a. Introduction

Two types of dam failures were considered in this study: "sunny-day" failure and "storm-day" failure.

A sunny-day failure typically is a piping failure. Piping is internal erosion of the embankment through displacement of fines by seepage. The erosion creates voids in the embankment and, therefore, could lead to breach and eventually collapse of the dam. It was assumed in this study that sunny-day failure occurs with minimal inflow to the reservoir and normal flow condition downstream.

A storm-day failure is associated with significant inflow into the impoundment. As a result of inadequate spillway capacity and reservoir storage capacity, overtopping of the embankment occurs. As the embankment is eroded, breach and ensuing failure develops.

b. Hydrology

To accommodate the storm-day dam-break analysis, inflow hydrographs for the reservoir resulting from a 100-year storm and four fractions (1, ¾, ½, ¼) of the probable maximum storm (PMS) were developed. Data necessary for generating the hydrographs include rainfall data and watershed characteristics.

The rainfall data for the 100-year storm were obtained from the National Weather Service's Rainfall Frequency Atlas of the United States Technical Paper 40 (Ref. 2) and HYDRO-35. To obtain a worst-case distribution, the rainfall data of 24-hour duration were critically arrayed such that the peak increment of rainfall occurred at the 12th hour proceeded by the second largest rainfall increment and followed by the third largest.

The rainfall data, or probable maximum precipitation (PMP) data, for estimating the probable maximum storm (PMS) which yields the maximum probable flood (PMF) were obtained from Hydrometeorological Report No. 51 (HMR51) (Ref. 3). The 72-hour duration rainfall data from HMR51 were processed according to the guidelines provided in Hydrometeorlogical Report No. 52 (HMR52) to give an estimated 24-hour duration rainfall distribution of the PMS (Ref. 4). This 24-hour duration rainfall was comprised of the four greatest 6-hour incremental rainfalls from the 72-hour duration rainfall data. The resulting total rainfall of the PMS for a 24-hour duration was calculated to be 29.8 inches.

The watershed model, HEC-1 (Ref. 5), was used to generate the inflow hydrographs resulting from the 100-year storm and the various fractions of PMS. Rainfall loss was assumed to be uniform at the rate of 0.05 inches per hour. The SCS unit hydrograph method was utilized in computing the hydrographs. This method requires the input of lag time. Based on surface condition, land slope, channel slope and flow length of the watershed, lag time was calculated as 1.5 hours.

The rainfall data and watershed characteristics were prepared by the Corps of Engineers and furnished to HWRE (Appendix 2). These data were then used by HWRE to develop inflow and outflow hydrographs in the subsequent analysis.

c. **Spillway Hydraulic Capacity**

A composite rating curve was developed for the principal and emergency spillways. The weir equation with a discharge coefficient of 2.6 was used to determine the flow rate. Flow which overtops the dam was also determined by the weir equation.

d. **Reservoir Routing**

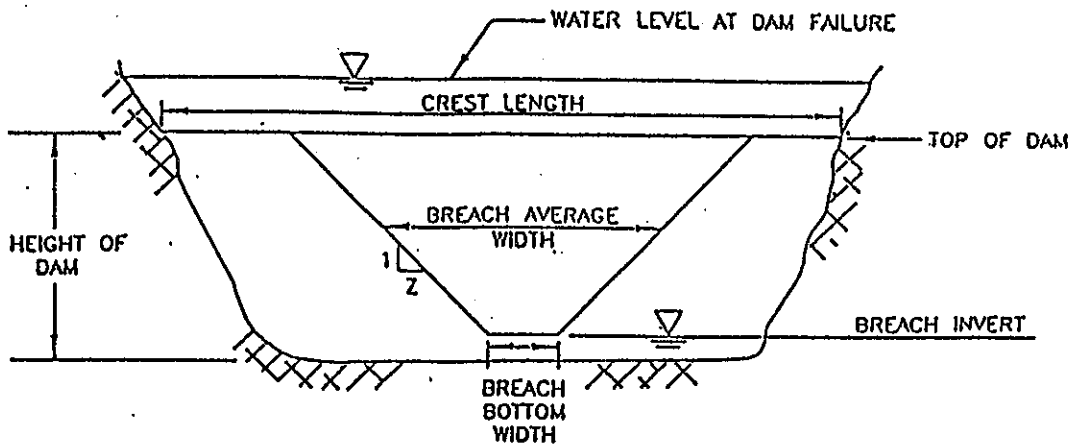
The inflow hydrographs were routed through the reservoir using the HEC-1 model to obtain outflow hydrographs. Information necessary for the reservoir routing includes elevation surface area relation of the reservoir, the composite spillway rating curve, dimensions and elevations of the dam. The dam-break flood forecasting model DAMBRK, a BOSS Corporation's enhanced version of the National Weather Service (NWS)'s model (Ref. 6), was also utilized to compute the reservoir outflow hydrographs for a check of the results from HEC-1. In the HEC-1 model, the Modified Puls Method is used to solve the continuity equation.

DAMBRK applies the dynamic wave method to flow routing in the downstream channel, thus requiring channel geometry and downstream boundary conditions.

The purpose of reservoir routing was to determine the inflow hydrograph which could be used as inflow condition for the hypothetical overtopping failure of the dam.

e. Breach Discharge Hydrograph

The discharge hydrograph of a breach is a function of the inflow hydrograph, reservoir storage and breach parameters (Ref. 6). The sketch below illustrates the various dam breach parameters for a typical earth-fill dam. Total outflow from the reservoir is a combination of flows through the breach, spillways and over dam crest, if any. As the breach in the dam develops, so does the breach discharge.



DEFINITION SKETCH OF BREACH PARAMETERS

f. Assumed Breach Parameters

Two of the parameters for a dam-break flood study are the average breach width "b" and breach time "t" (time from the beginning to full formation of breach). Fread (Ref. 6) has developed two equations to estimate these two parameters. For the current dam, the equations yield b= 70 feet, t= 0.24 hours for overtopping failure, and b= 50 feet, t= 0.24 hours for piping failure (Appendix 3). Experience shows that b≈ 3 h_d, where h_d= dam height. Chester Dam has a height of 20 feet. Therefore an average of b= 60 feet seems reasonable for both failure scenarios. The breach time of 0.24 hours appears too short. Instead t= 0.5 hours was selected. The breach discharges computed by DAMBRK using b= 60 feet and t= 0.5 hours agree fairly well with that given by another equation developed by Fread for checking the parameters (Appendix 3).

It was assumed that the breach for overtopping failure was of trapezoidal shape with side slopes of 1H: 1V. Therefore the breach bottom width was 40 feet. The shape of the breach for piping failure was assumed to be rectangular because the breach normally develops from within the embankment. These and other parameters necessary for the dam-break flood studies are listed below:

Assumed Sunny-Day (Piping) Failure Condition:

i) Initial pool level: 1062.6 feet NGVD

ii) Dam failure level (water surface that triggers beginning of breach): 1062.6 feet NGVD

iii) Breach invert elevation: 1045 feet NGVD

iv) Breach bottom width: 60 feet with side slopes 1 V : 0 H

v) Time to complete formation of breach: 0.5 hours

vi) Downstream reach roughness (Manning's n values):

0.050 to 0.100 for channel
0.040 to 0.100 for overbank

vii) Embankment dimensions:

Height of dam = 20 feet
Crest length = 220 feet

Assumed Storm-Day (Overtopping) Failure Condition:

i) Initial pool level: 1062.6 feet NGVD

ii) Dam failure level (water surface that triggers beginning of breach): 1066.8 feet NGVD or 1.8 feet above top of dam

iii) Breach invert elevation: 1045.0 feet NGVD

iv) Breach bottom width: 40 feet with side slope 1 V : 1 H

v) Time to complete formation of breach: 0.5 hour

vi) Downstream reach roughness (Manning's n values):

0.050 to 0.100 for channel
0.040 to 0.100 for overbank

vii) Embankment dimensions:

Height of dam = 20 feet
Crest length = 220 feet

g. Downstream Channel Routing

Downstream channel routing was performed using the DAMBRK model. A downstream channel routing analysis allows the breach discharge hydrograph to be characterized at points of interest along the stream. The breach discharge is attenuated and stored through a downstream channel and flood plain in a manner similar to that by which an inflow hydrograph is routed through a reservoir. The degree of attenuation of this breach discharge hydrograph is a function of downstream valley storage capacity and valley roughness characteristics.

(1) Method

The dynamic wave method of channel routing is used in DAMBRK to route the flood wave downstream. This is a hydraulic routing method that solves the complete equations for unsteady flow. Output from the computer code includes flood discharge, stage, and their timing at various locations along the channel.

(2) Downstream Cross-Sections

Cross-sections of the study reach were obtained from field survey by HWRE (Appendix 1) with locations shown in Fig. 1. USGS topographic maps were used to supplement HWRE's survey data. Manning's "n" values for the channel and overbanks were determined based on the size of channel bed material and vegetation condition (Ref. 7). These values are listed in the previous section.

(3) Downstream Flow Structures

The culverts on the stream leading from the reservoir are expected to cause some restriction to flood flows. However, because the channel is very steep and the valley is very narrow, this effect is expected to be limited to the vicinity of the structures only. If these structures should be considered, they have to be treated as bridges in the DAMBRK model. Attempts were made to include all culverts in the model, but no convergent solutions were given by DAMBRK. Usually, the program broke down. This difficulty is attributed to the particular flow situation, i.e., mixed flows. Because of the co-existence of supercritical flow and subcritical flow, flow becomes unstable at sections where Froude number approaches unity (critical flow). Considering their limited effect, the difficulty in obtaining a stable and reasonable computer solution, and the fact that there are no other structures nearby, the first four culverts (upstream of Route 11) were not included in the simulation.

The culvert passing under Route 11 was considered in the flood routing. Hydraulic capacity of the culvert was estimated to be about 100 cfs as water surface reaches the top of the road surface. Any significant flood such as those used for the present dam-break study would overtop the road. Neglecting the culvert and using the valley cross-section for the flood routing would result in flow stages that are too low. Therefore, a rating curve was developed for the culvert based on pipe-full flow condition (Appendix 3). For flows below top of the culvert, discharges were determined by linear interpolation. The capacity of the culvert is at least one-order smaller than the flood peak discharge. Errors in estimating the flow through the culvert would not produce any significant effect on the routing results because the majority of the flow is conveyed over the top of the road as weir flow. Weir equation was used to determine the flow over the road. The road embankment was treated as a dam with a length of 200 feet. A discharge coefficient of 3.0 was assumed.

The bridge over Middle Branch Williams River 1.61 miles downstream from the dam does not restrict flow in the channel because the bridge abutments were set at the edges of the channel. The bridge was therefore not considered as an input in the analysis. The channel cross-section at the bridge was, however, included in the input data to define stream geometry at that location.

In the routing procedure, flow structures below Chester Dam were assumed to have full hydraulic capacity. If the structures become blocked with debris, the peak water surface upstream could increase to stages higher than estimated. In addition, to estimate the maximum water level, the embankment of Route 11 was assumed not to fail. However, because of the increased flood stages and velocities associated with a dam-break, failure of any or all of the flow structures is possible. This study does not attempt to predict if any downstream structure will fail during failure of Chester Dam.

(4) Antecedent Channel Flow

Under normal conditions, flow carried by the reservoir stream is no more than a few cubic feet per second. Initially, a flow of 10 cfs (minimum required by DAMBRK) was tested. The flow was then increased until the program converged. It was found that, for both sunny-day and storm-day failure scenarios, an initial flow of 50 cfs was needed and, therefore, used in the simulations. Since the magnitude of this flow rate is less than 3% of the peak flood flows, any effect on the routing results due to the initial flow would be negligible. The initial flow in Middle Branch Williams River was assumed to be 100 cfs based on the fact that its drainage area is over 30 times larger than the drainage area of Chester Reservoir.

h. Lateral Flow

At the confluence of the reservoir stream and Middle Branch Williams River, flow from Middle Branch Williams River was treated as lateral flow. For sunny-day scenario, a constant lateral flow of 100 cfs was assumed. For storm-day scenario, a hydrograph with the same occurrence frequency as that (which was later determined to be 3/4 PMF) of the inflow hydrograph for the reservoir was used. To simplify the problem, a triangular hydrograph was assumed. The derivation of this hydrograph is included in Appendix 2.

i. Calibration

Before the simulation of a dam-break flood, the program (DAMBRK) should be calibrated. For a gaged stream, a rating curve is the ideal data for the calibration. This type of information is, however, not available for the reach of the stream under study. The calibration for this study was done by routing the PMF through the stream and comparing the range of inundation with elevation contours in the USGS topographic map. Necessary adjustments in roughness, location and geometry of cross-sections were made until reasonable agreement was reached.

j. Project Mapping

The project mapping was developed by enlarging the USGS 1:24,000 Metric Quadrangle (7.5 x 15 minute) of Chester, Vermont. Locations of structures within the inundation limits were verified through field survey and site reconnaissance.

k. Vertical Control

Vertical Control for this investigation was established from a level loop on a standard USGS disc stamped " P 32 1942". The disc is set horizontally in a large boulder located near the north end of a bank cut along Route 11, 1.1 miles northwest from the public school building in Chester. It is 34.0 feet northeast of road centerline, 14.5 feet north of a pipe culvert. A level run was also done starting at this same disc and ending at a standard disc stamped " N 32 1942" set vertically in the west end of the south face of the school building.

5. RESULTS OF ANALYSIS

a. Inflow Hydrograph

The results of reservoir routing using HEC-1 are summarized in Table 1. The complete computer output is included in Appendices 4 and 5. Flow hydrographs for the 100-year storm and the four fractions of PMF are shown in Fig. 3. It is seen that the inflow hydrograph resulting from the 100-year storm peaks at 14 hours after beginning of the storm with a peak discharge of 400 cfs. The PMF inflow hydrograph peaks also at 14 hours but has a peak discharge of 2,270 cfs. Since the reservoir storage is very small, outflow hydrographs are almost identical to the inflow hydrographs. In general, the difference between peak inflow and peak outflow is less than 1%. The results from DAMBRK reservoir routing (without dam-failure) show that the computed peak outflows are generally about 2% smaller than those from HEC-1. This difference is insignificant. The results from both HEC-1 and DAMBRK are therefore considered to be reasonable.

As seen in Table 1, all the inflow hydrographs resulted in flow overtopping the dam. Water depth above the dam crest varies from 0.6 feet for the 100-year storm to 2.5 feet for the full PMF. The 3/4 PMF yielded a water stage 2 feet above the dam crest. According to experience and recommendation in the scope of work for this study, overtopping failure was considered to occur when an inflow hydrograph results in a water level no more than a few feet above dam crest. The 3/4 PMF inflow hydrograph was therefore selected to be the inflow condition for storm-day failure analysis.

b. **Reservoir Storage Capacity**

The maximum storage capacity of the reservoir, i.e., storage at dam crest, is approximately 35 acre-feet calculated by HEC-1. The calculated storage is less than original design storage (46 acre-feet) primarily because the method in reservoir routing treats the reservoir storage as an inverse cone. However, it should be pointed out that continuous deposition in the reservoir over the years since construction of the dam is expected to have reduced the storage. The calculated storage is probably more realistic. As the 3/4 PMF outflow reaches its peak stage of 1067.0 feet, the volume of water stored in the reservoir is calculated to be 46 acre-feet.

c. **Spillway Hydraulic Capacity**

The computation shows that the combined maximum hydraulic capacity for the principal and emergency spillways is approximately 105 cfs, about 30% less than the reported design capacity (155 cfs). It is obvious that Chester Reservoir does not have adequate storage and spillway capacities to route and pass any of the floods analyzed in this study.

d. **Breach Discharge Hydrograph**

Tables 2 and 3 summarize the peak discharges and stages at critical stations along the downstream channel due to sunny-day and storm-day failures, respectively. The discharge and stage hydrographs at these stations are shown in Figs. 4 and 5. Complete computer output for each of the two scenarios is included in Appendices 6 and 7 respectively. Sunny-day failure was assumed to start at 0.0 hour. A peak flow of 2000 cfs was produced at 0.23 hours due to failure of the dam. At Route 11, peak flow occurred at 0.33 hours and was reduced to 1,860 cfs. The reduction is small because the valley has virtually no storage upstream from Route 11. At the end of the river reach under study, peak flow was reduced to 1,570 cfs.

The storm-day dam failure resulted in a peak flow of 4,140 cfs at the dam 14.03 hours after the beginning of the storm, or, 0.33 hours after breach started. The peak flow was slightly reduced to 3,930 cfs at Route 11, and it occurred at 14.55 hours, or, 0.85 hours after breach started. Compared to the flow contributed from Middle Branch Williams River drainage area upstream of its confluence with the reservoir stream, the dam-failure flood from Chester Reservoir is small (one order smaller). As seen in Fig. 5, it causes only small peaks in the hydrographs at 1.61-mile and 2.56-mile stations. For comparison, the routing results without dam-failure assumed are presented in Table 4. It is seen that failure of Chester Dam caused less than 7% increase in peak flow in Middle Branch Williams River. The major portion of the flood flow is carried by the river from its drainage area upstream from the confluence with the reservoir stream.

e. **Downstream Channel Routing**

One of the major parameters which define the severity of a flood is the flood stage. The peak flood profiles resulting from the two hypothetical dam-break floods at the surveyed cross-sections along the stream under study are depicted in Figs. 6 and 7 , respectively. Flow conditions at critical locations are described below (all elevations are referred to NGVD).

(1) Sunny-Day Results

At the reservoir road crossing (0.32 mile), peak flow and peak stage are 1,940 cfs and 959.4 feet respectively. Water surface is below top (EL. 961.7 feet) of the road running parallel with the stream. The culvert (4 feet wide x 8 feet high) located upstream of this cross-section has an estimated maximum discharge capacity of 500 cfs based on pipe-full flow condition and, therefore, is expected to be a restriction.

Peak flow and peak stage at the cross-section at Route 11 are 1,860 cfs and 715.7 feet respectively. Because the culvert under the road has a maximum capacity of less than 100 cfs, most of the flow passed over the road, resulting in a water depth of about 2 feet above road surface (EL. 713.6 feet).

At the road crossing over Middle Branch Williams River (1.61 miles), peak flow is 1,790 cfs which includes a lateral inflow of 100 cfs from Middle Branch Williams River. Peak stage at this section is 658.4 feet, or, about 13 feet below top of road (EL. 671.5 feet).

At the end of the study reach (2.56 miles), peak flow and peak stage are 1,570 cfs and 604.6 feet, respectively. Water surface is 7 feet below top

(EL. 612.0 feet) of the left bank. As in other cross-sections along Middle Branch Williams River, the dam-break flood flow is confined in the river channel.

(2) Storm-Day Results

The storm-day failure results in a peak flow of 4,090 cfs at the reservoir road crossing. Peak stage is 962.6 feet and overtops the road parallel with the stream by 1 foot. The culvert upstream is expected to be overtopped because this flood flow is much larger than the sunny-day flood which is predicted to exceed the culvert capacity.

At Route 11, peak flow and peak stage are 3,930 cfs and 717.0 feet, respectively. The flood overtops the road by 3.5 feet, an increase of 1.5 feet over the flood stage due to sunny-day failure.

At the road crossing over Middle Branch Williams River, peak flow and peak stage are 39,300 cfs and 673.9 feet, respectively. The flow overtops the bridge by about 2.4 feet. It should be pointed out, however, that the peak flow and stage at this section and other sections along Middle Branch Williams River are mainly due to the flood flows from Middle Branch Williams River drainage area rather than dam-break flood from Chester Reservoir. As shown in Table 4, the routing without dam-failure assumed yields a stage of 673.6 feet at this section. The dam-break flood increased water level by only about 0.3 feet.

At the end of the study reach, peak flow and stage are 38,900 cfs and 617.8 feet, respectively. Water surface is about 6 feet above top of the left bank and about 3 feet above the street leading to the foot bridge. Again, the flood at this location is mainly due to the flood flow carried by Middle Branch Williams River.

f. Inundation Mapping

The limits of inundation caused by the two hypothetical dam failure floods were estimated base on the maximum computed stages along the downstream channel. The flooded area resulting from the sunny-day failure is depicted in Fig. 8 and is predicted to inundate the houses and armory near the section where the reservoir stream passes Route 11. Route 11 would be overtopped by the flood by 2 feet. Downstream in Middle Branch Williams River, the flood flow is confined in the channel. No overbank flow is predicted.

Fig. 9 shows the inundation limit of the storm-day failure flood. This flood is predicted to overtop Route 11 by 3.5 feet. All the houses located along Route 11 on the left bank would be inundated. As discussed earlier, the maximum flood levels along the Middle Branch Williams River are caused primarily by flood flow carried by the river from the upstream watershed rather than from Chester Reservoir. Structures and houses directly affected by the dam-break flood from Chester Reservoir are those located upstream and immediately downstream of Route 11 crossing at 0.91 miles.

g. **Size Classification**

Chester Dam is 20 feet high and its design maximum storage is 46 acre-feet. According to Article 2.1.1 of the "Recommended Guidelines for Safety Inspection of Dams" (Ref 1), the dam is classified **SMALL** in size.

h. **Hazard Classification**

This analysis shows that the flood due to a sunny-day failure would inundate several houses upstream and immediately downstream from Route 11 crossing. Route 11 would be overtopped. The storm-day failure flood would result in a much larger inundation area. The houses and buildings along Route 11 in the town of Chester would be inundated. On the basis of its potential to cause downstream damage, in terms of either loss of life or economic loss, Chester Dam is rated Class 2 or a **SIGNIFICANT** hazard category.

TABLE 1

RESERVOIR ROUTING SUMMARY
(HEC-1 Model Results)

Flood Frequency	Peak Inflow (cfs)	Peak Outflow (cfs)	Peak Stage (ft. NGVD)	Water Depth above Dam Crest (ft)	Flow Condition
100-year	400	385	1065.6	0.6	overtopped
1/4 PMF	568	564	1065.9	0.9	overtopped
1/2 PMF	1136	1136	1066.5	1.5	overtopped
3/4 PMF	1704	1695	1067.0	2.0	overtopped
1 PMF	2271	2266	1067.5	2.5	overtopped

TABLE 2

DOWNSTREAM CHANNEL ROUTING RESULTS
for Sunny-Day Failure

Downstream Location	Peak Discharge (cfs)	Peak Stage (ft NGVD)	Depth Above Streambed (ft)	Time to Peak Stage After Breach (hours)
Chester Dam (at 0.0 mi.)	2005	1062.7	17.7	0.03
Reservoir Road (at 0.32 mi.)	1940	959.4	8.3	0.28
Route 11 (at 0.91 mi.)	1860	715.7	7.7	0.33
Road Crossing (at 1.61 mi.)	1790*	658.8	6.0	0.43
Foot Bridge (at 2.56 mi.)	1570	604.6	4.2	0.58

* include lateral inflow of 100 cfs

TABLE 3

DOWNSTREAM CHANNEL ROUTING RESULTS
for Storm-Day Failure

Downstream Location	Peak Discharge	Peak Stage	Depth Above Streambed	Time to Peak Stage After Start of Storm	Time to Peak Stage After Start of Breach**
	(cfs)	(ft NGVD)	(ft)	(hours)	(hours)
Chester Dam (at 0.0 mi.)	4140	1067.0	20.0	14.03	0.33
Reservoir Road (at 0.32 mi.)	4090	962.6	11.5	14.43	0.73
Route 11 (at 0.91 mi.)	3930	717.0	9.0	14.55	0.85
Road Crossing (at 1.61 mi.)	39300*	673.9	21.1	14.63	0.93
Foot Bridge (at 2.56 mi.)	39000	617.8	17.4	14.70	1.00

* Includes lateral inflow
** Dam begins to breach at 13.70 hrs when the reservoir water surface reaches 1066.8 feet NGVD.

TABLE 4

DOWNSTREAM CHANNEL ROUTING RESULTS
for Storm-Day without Dam Failure

Downstream Location	Peak Discharge (cfs)	Peak Stage (ft NGVD)	Depth Above Streambed (ft)
Chester Dam (at 0.0 mi.)	1656	1067.0	2.0
Reservoir Road (at 0.32 mi.)	1635	958.8	7.7
Route 11 (at 0.91 mi.)	1606	715.4	8.5
Road Crossing (at 1.61 mi.)	37389	673.6	20.8
Foot Bridge (at 2.56 mi.)	37350	617.6	17.1

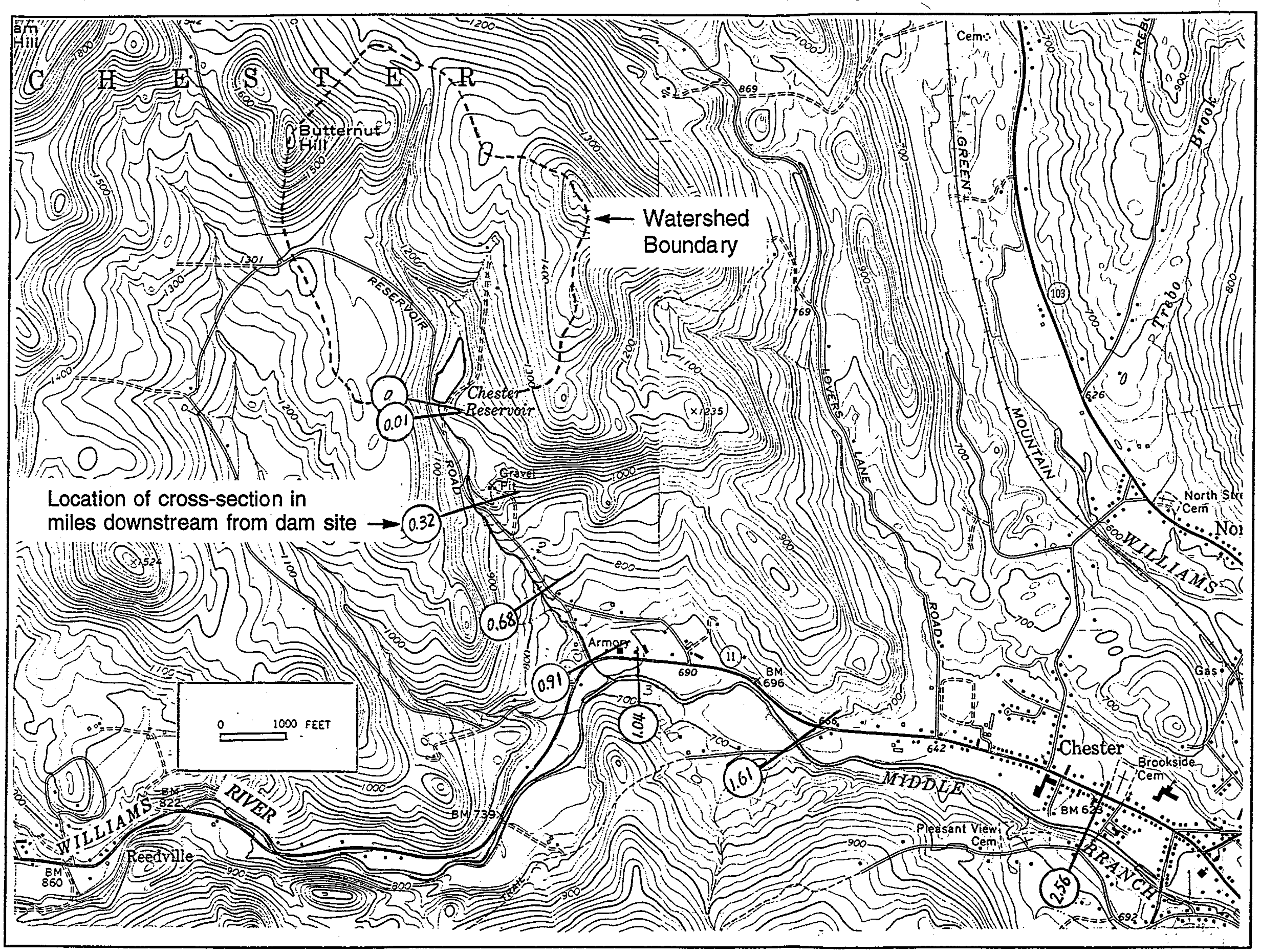


Figure 1. Index Map

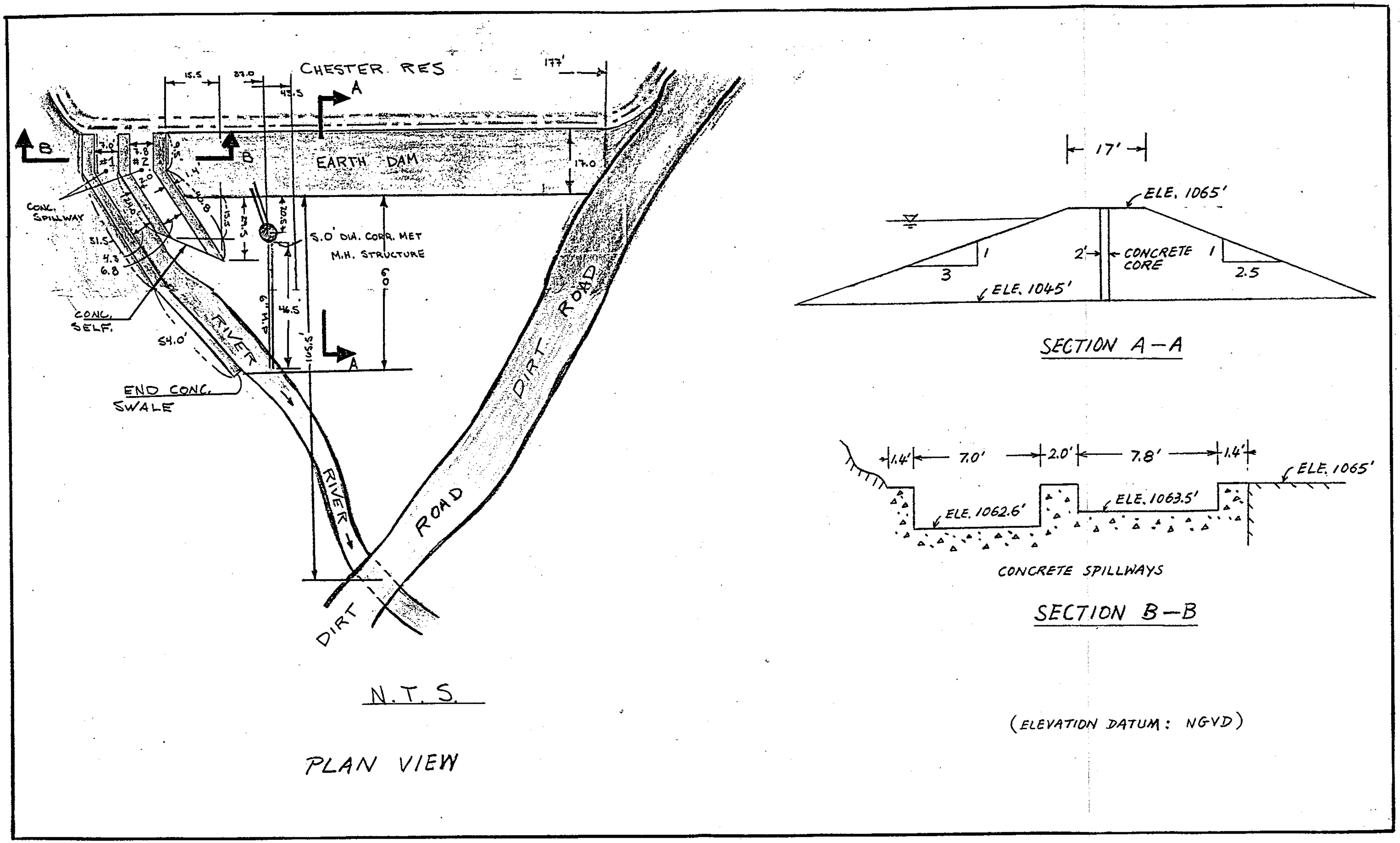


Figure 2. Sckematic of Chester Dam

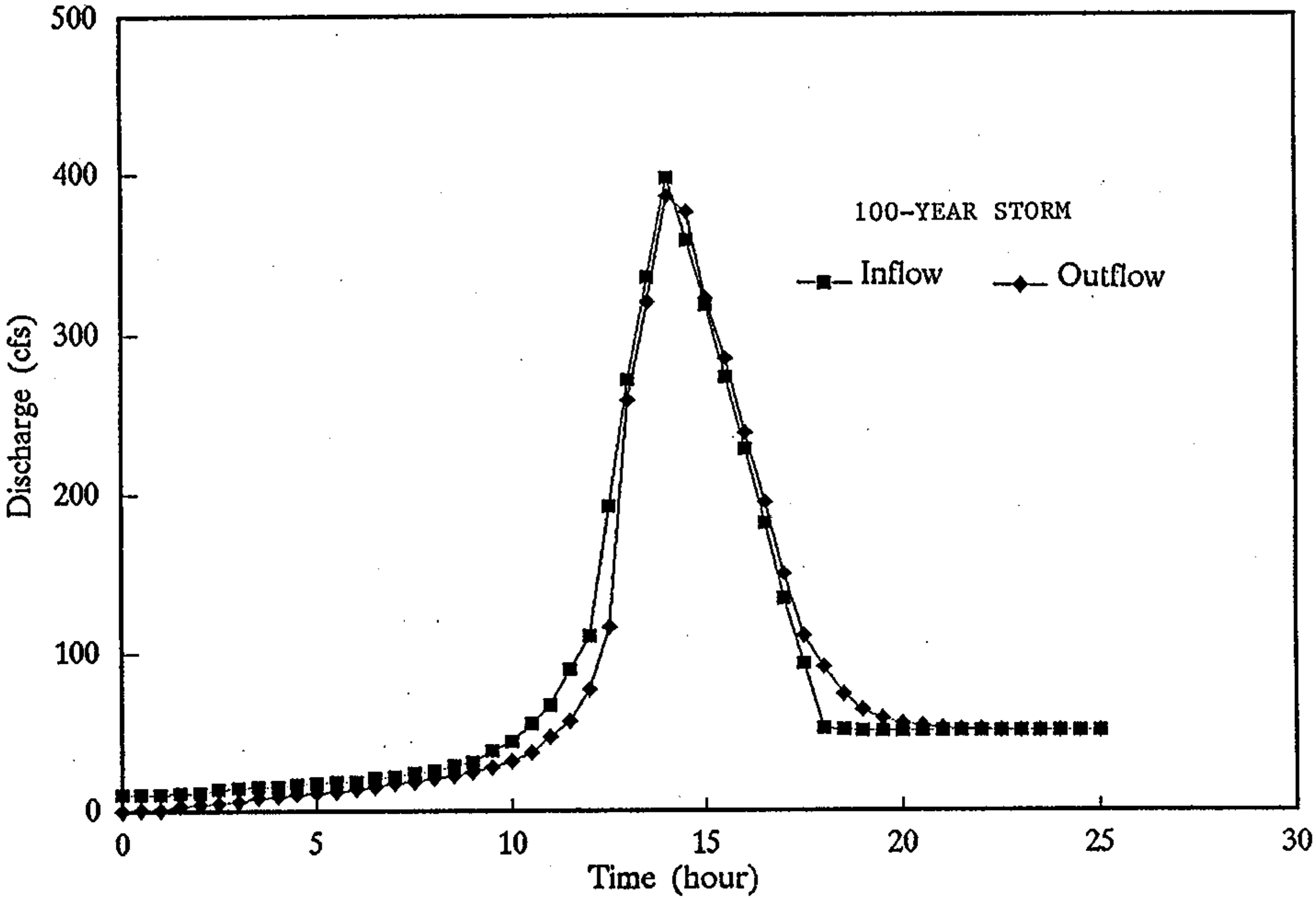
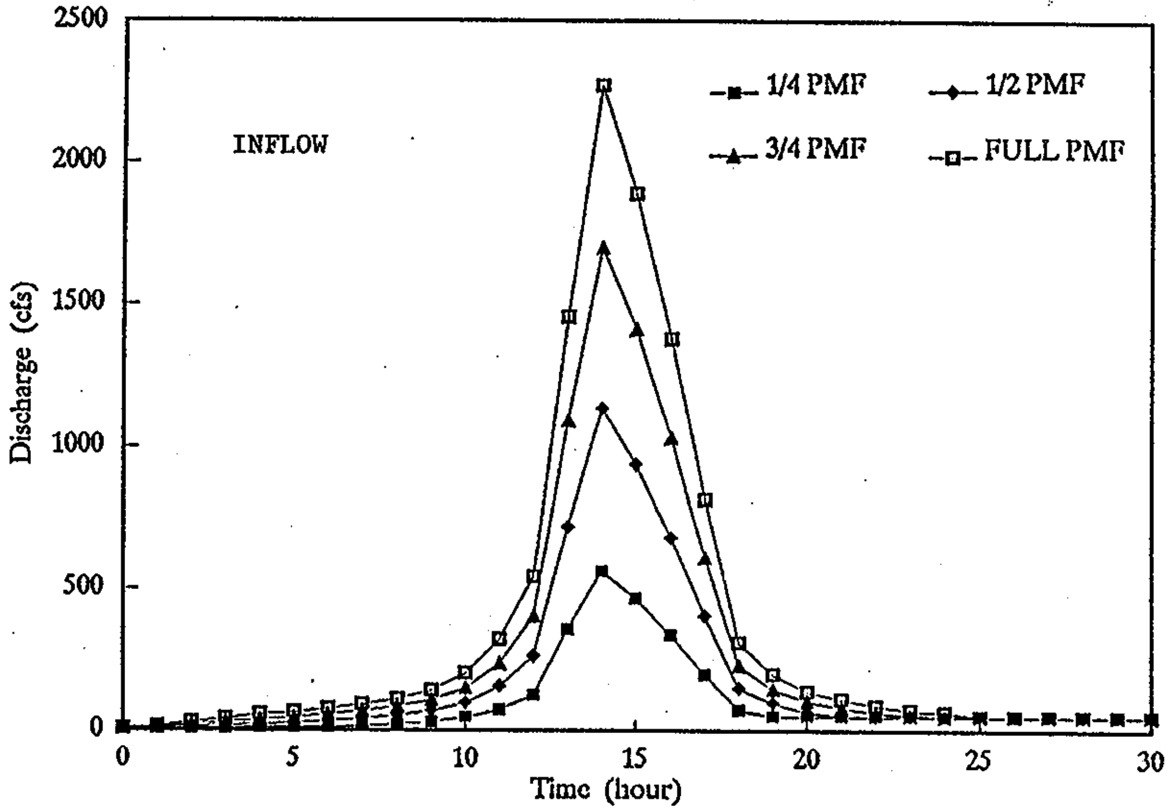


Figure 3. Inflow and Outflow Hydrographs (Reservoir Routing)



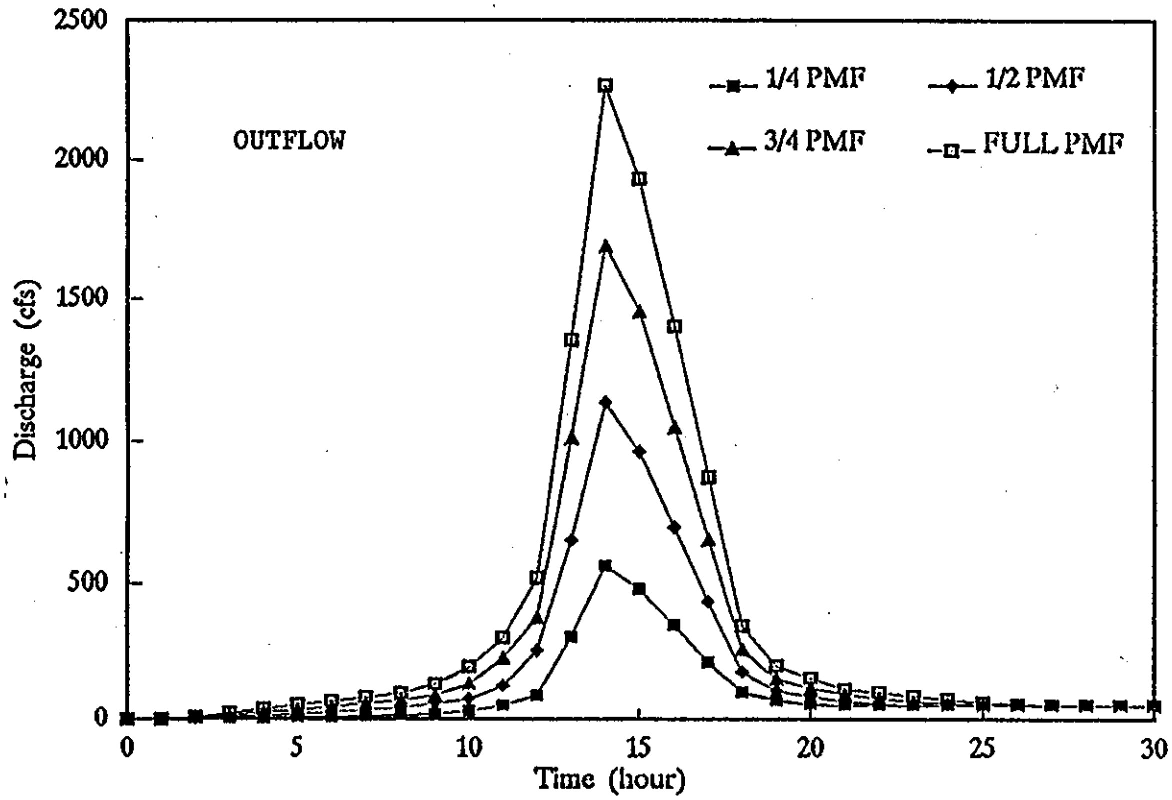


Figure 3. (continued)

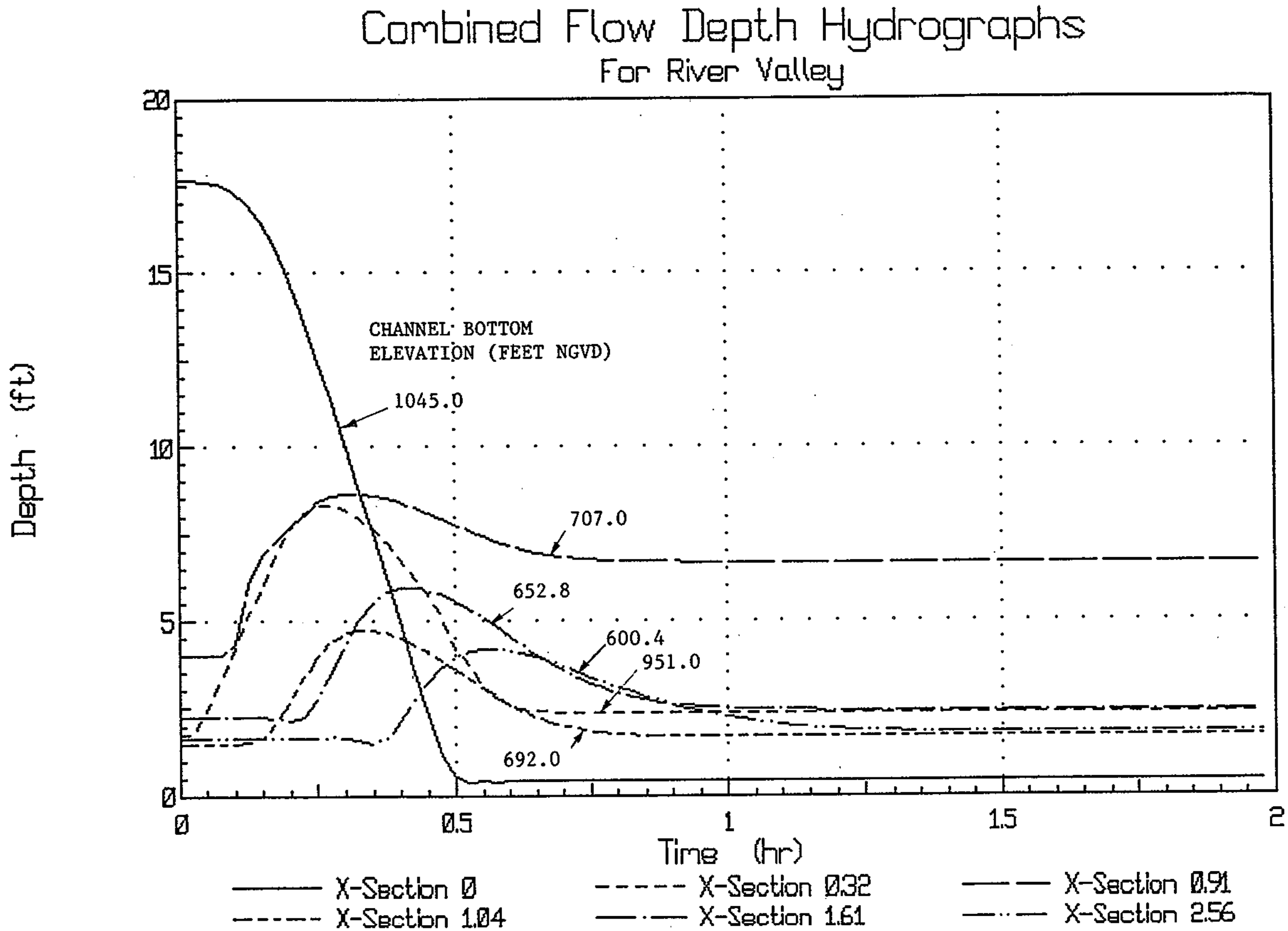
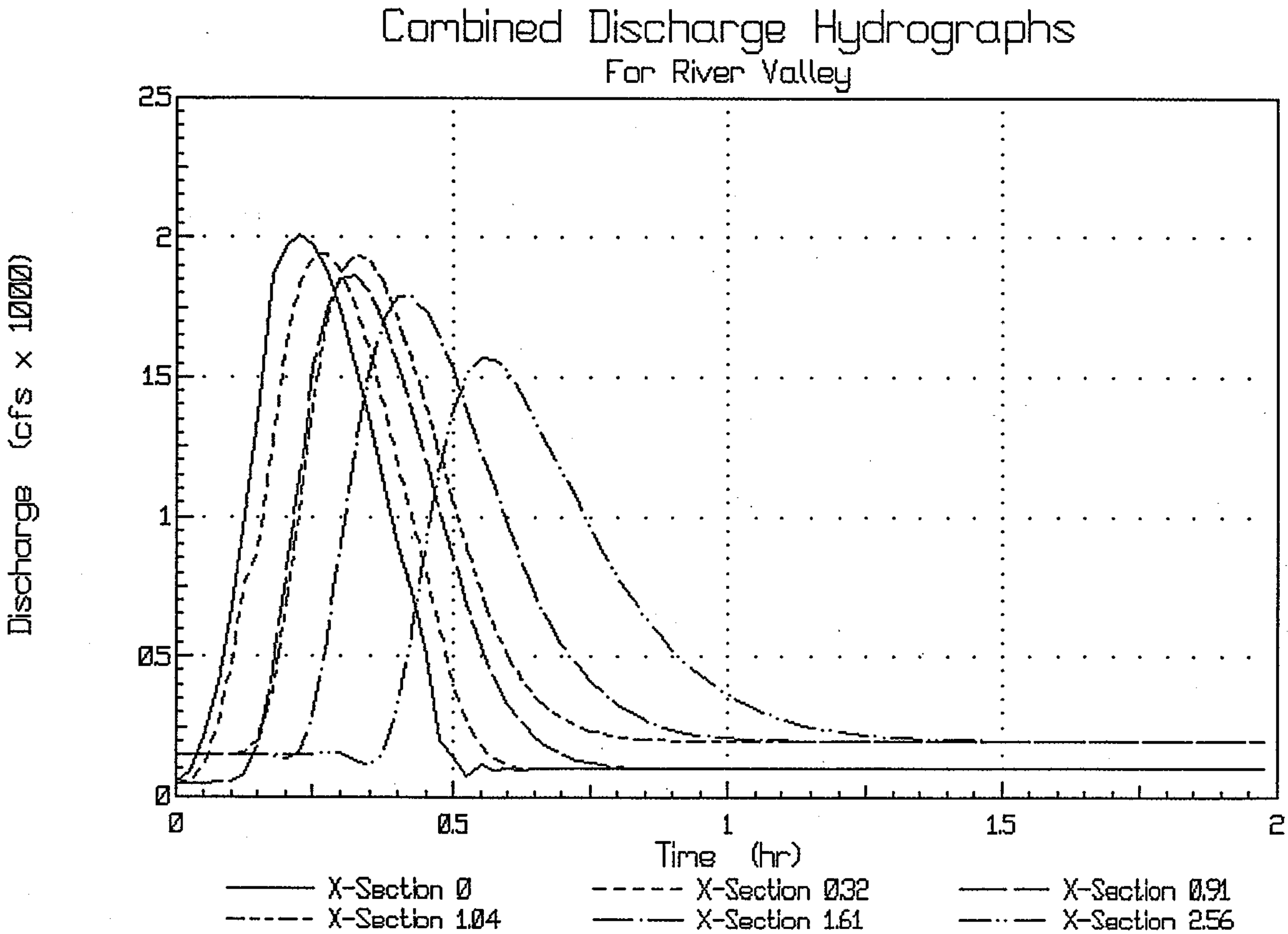


Figure 4. Stage and Flow Hydrographs Resulting from Sunny-Day Dam-Break Flood.



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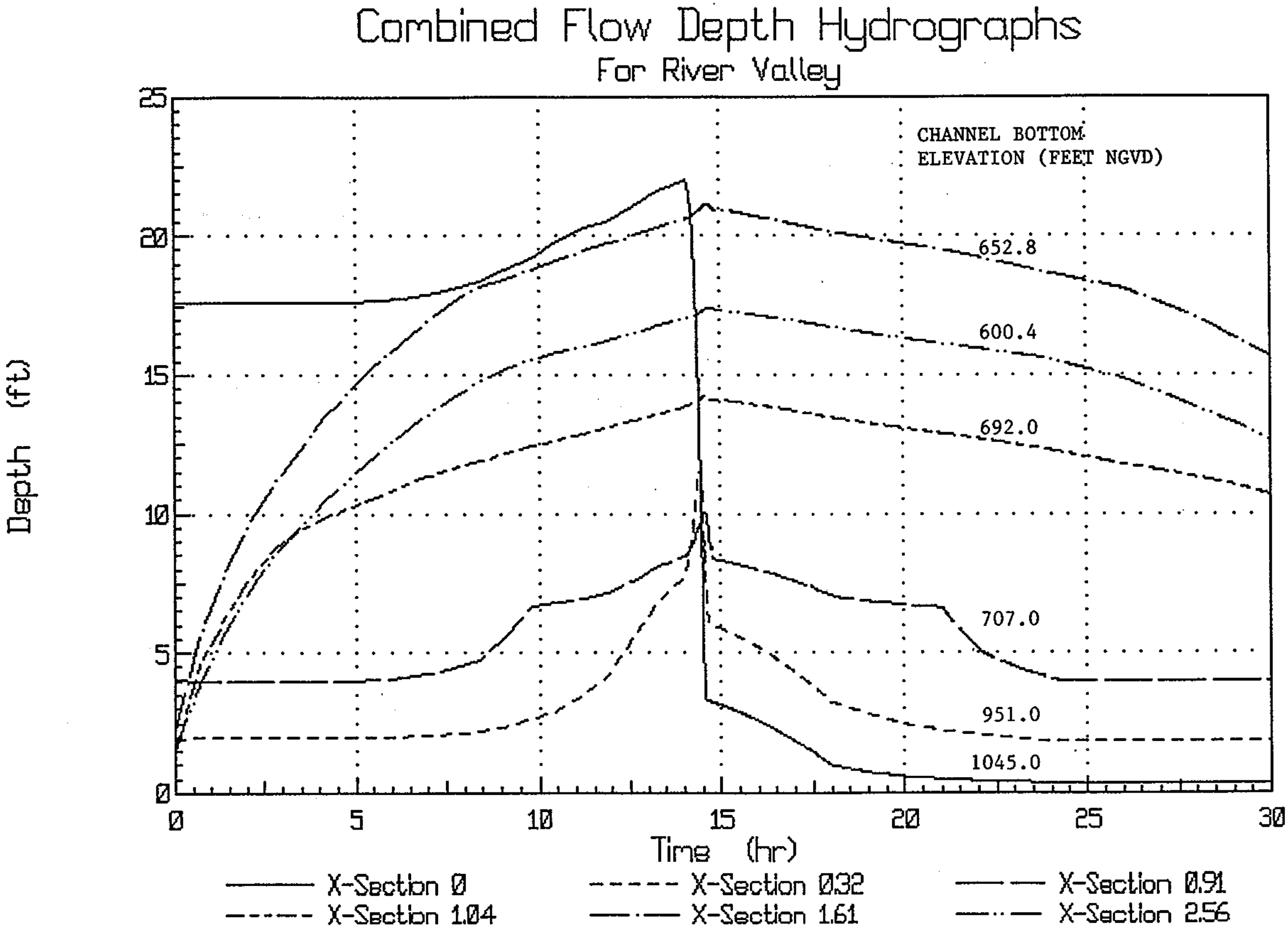
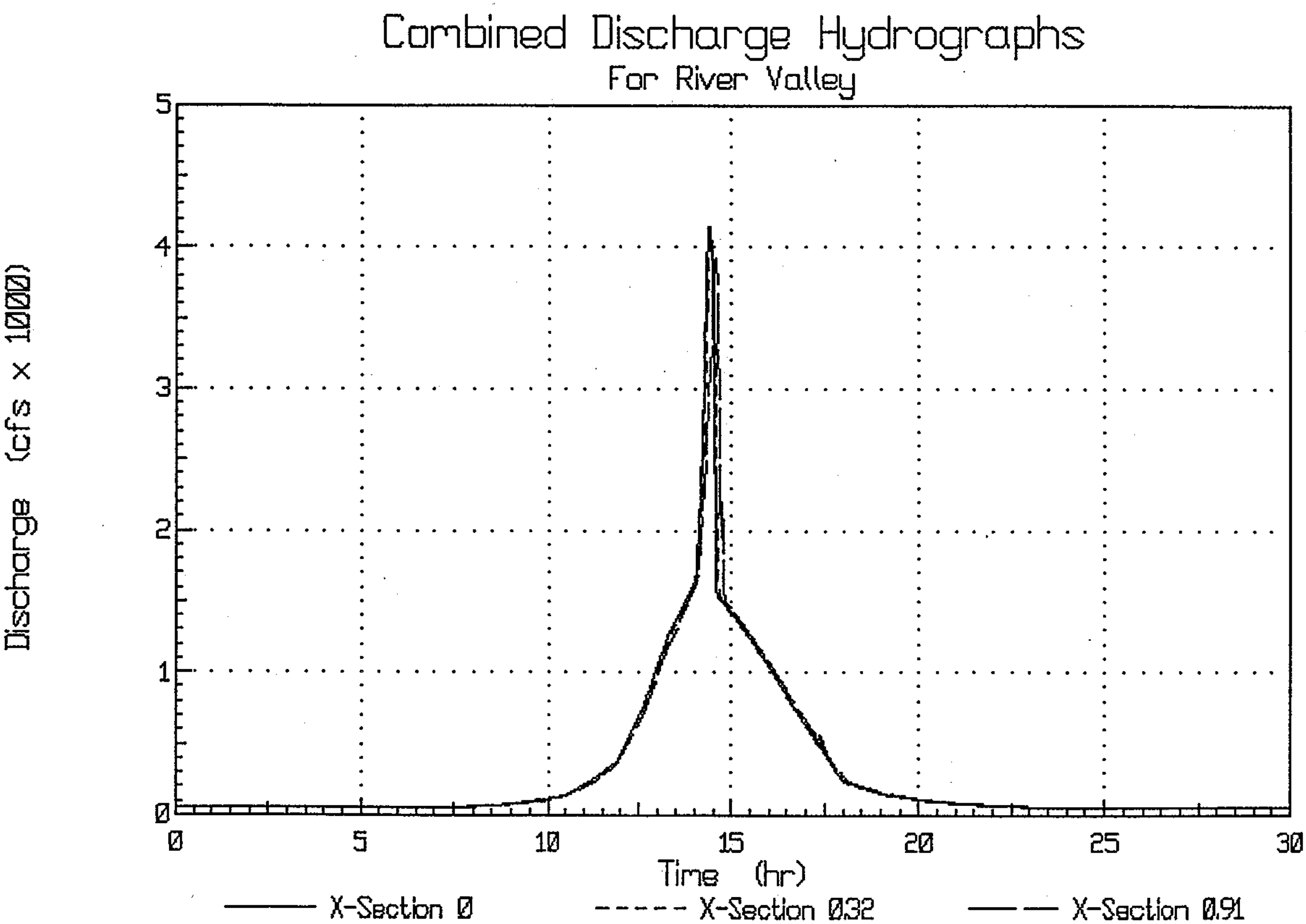


Figure 5. Stage and Flow Hydrographs Resulting from Storm-Day Dam-Break Flood.



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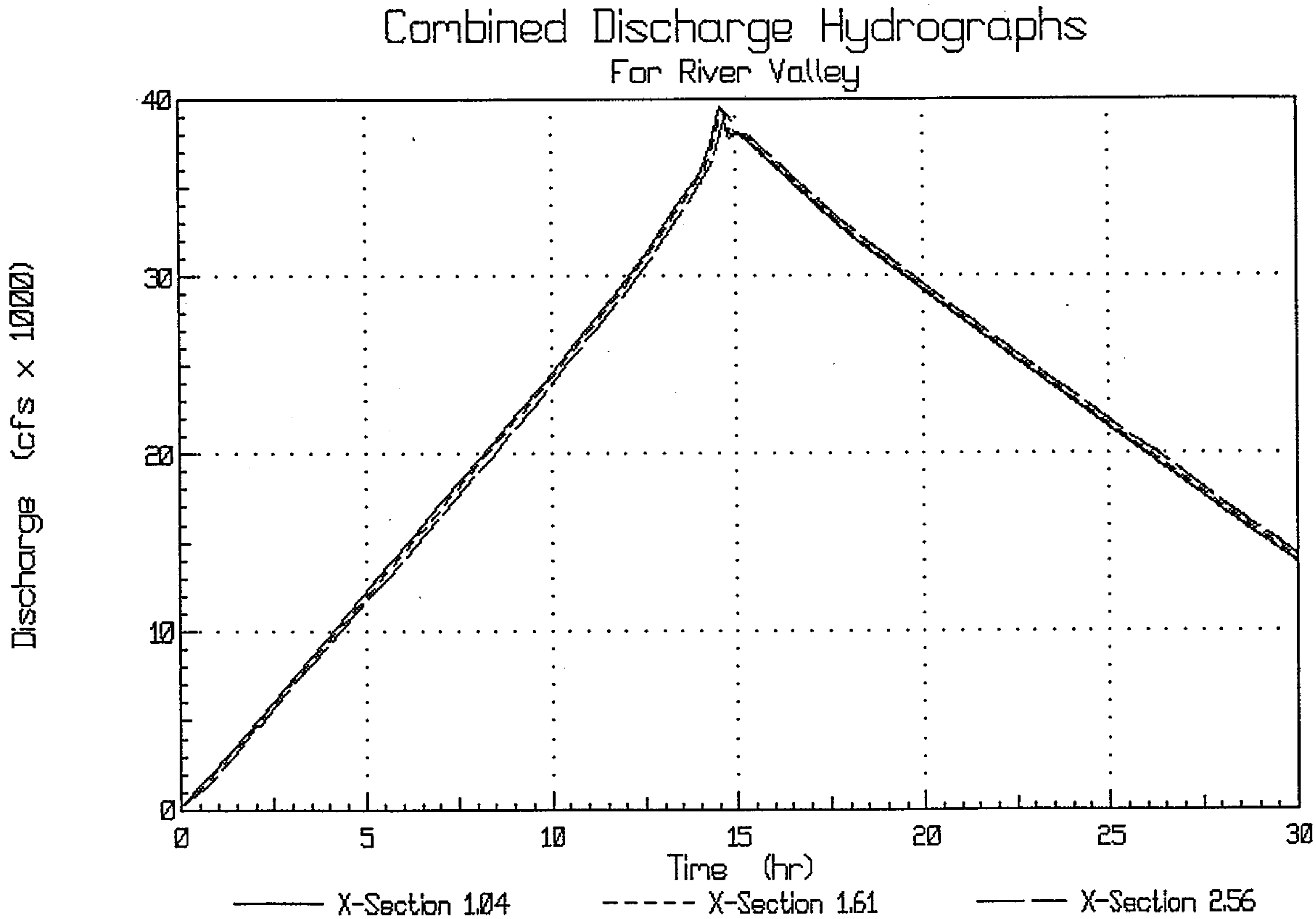

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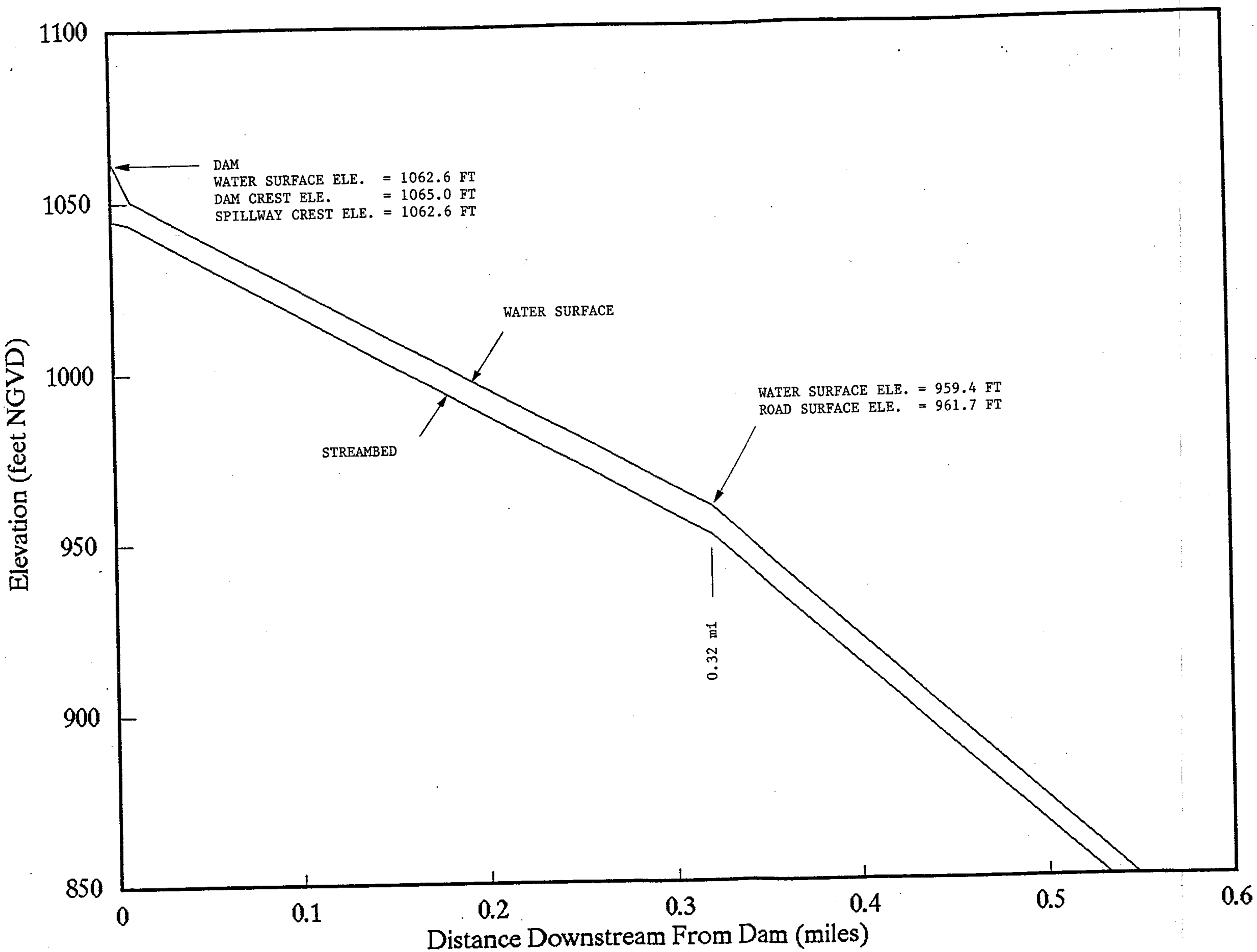
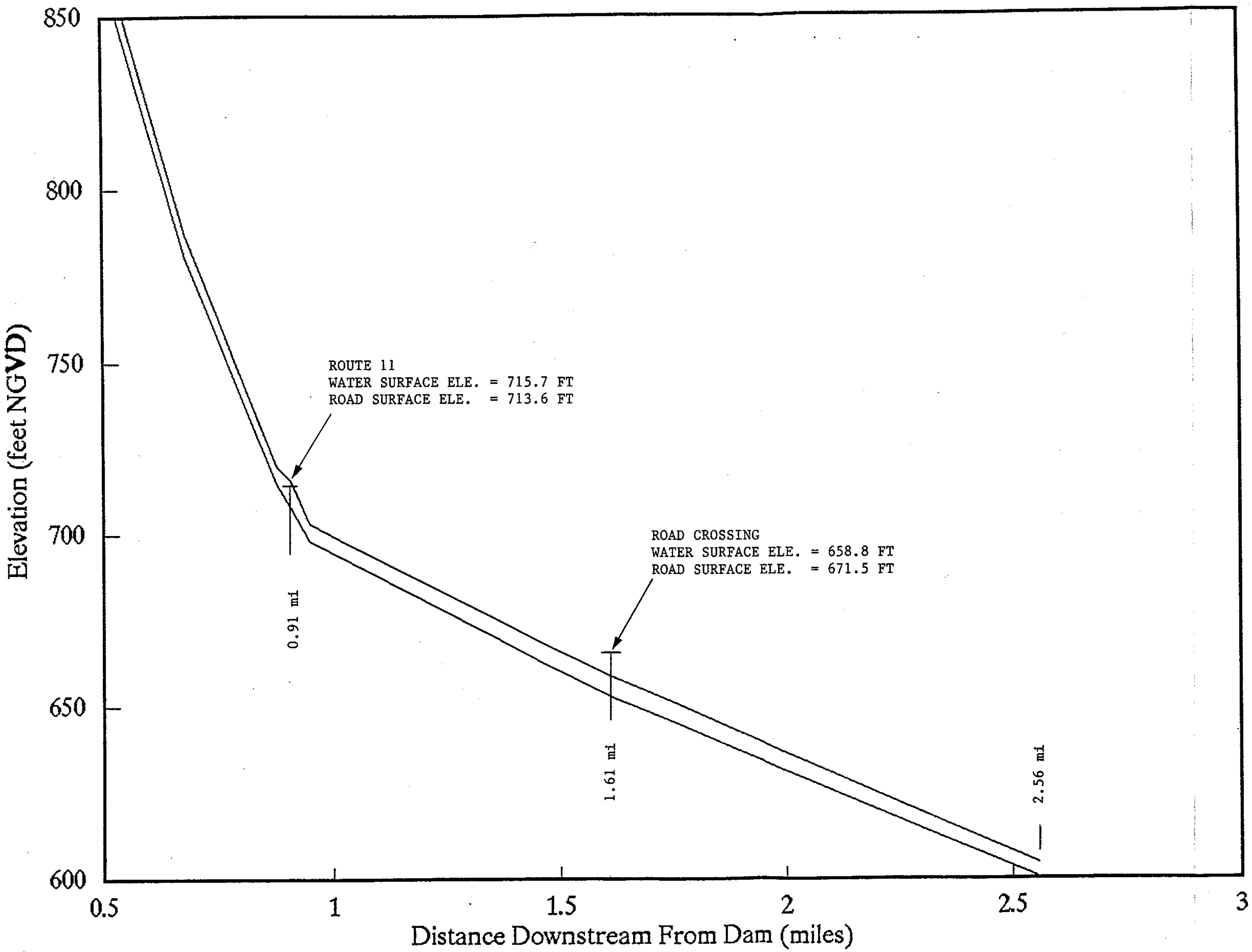


Figure 6. Peak Flood Stages along the Downstream Channel for Sunny-Day Failure Scenario.



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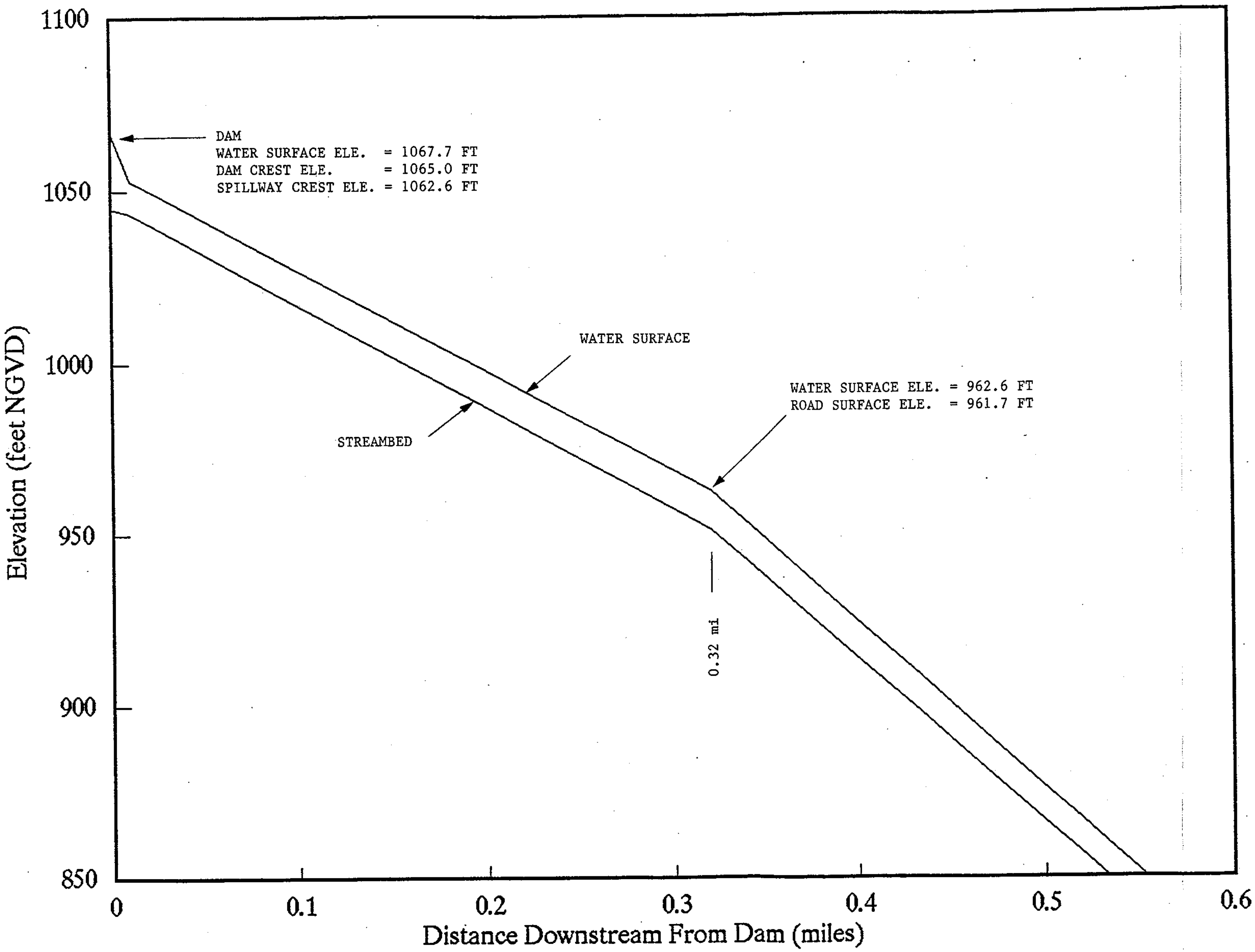

Figure 7. Peak Flood Stages along the Downstream Channel for Storm-Day Failure Scenario.

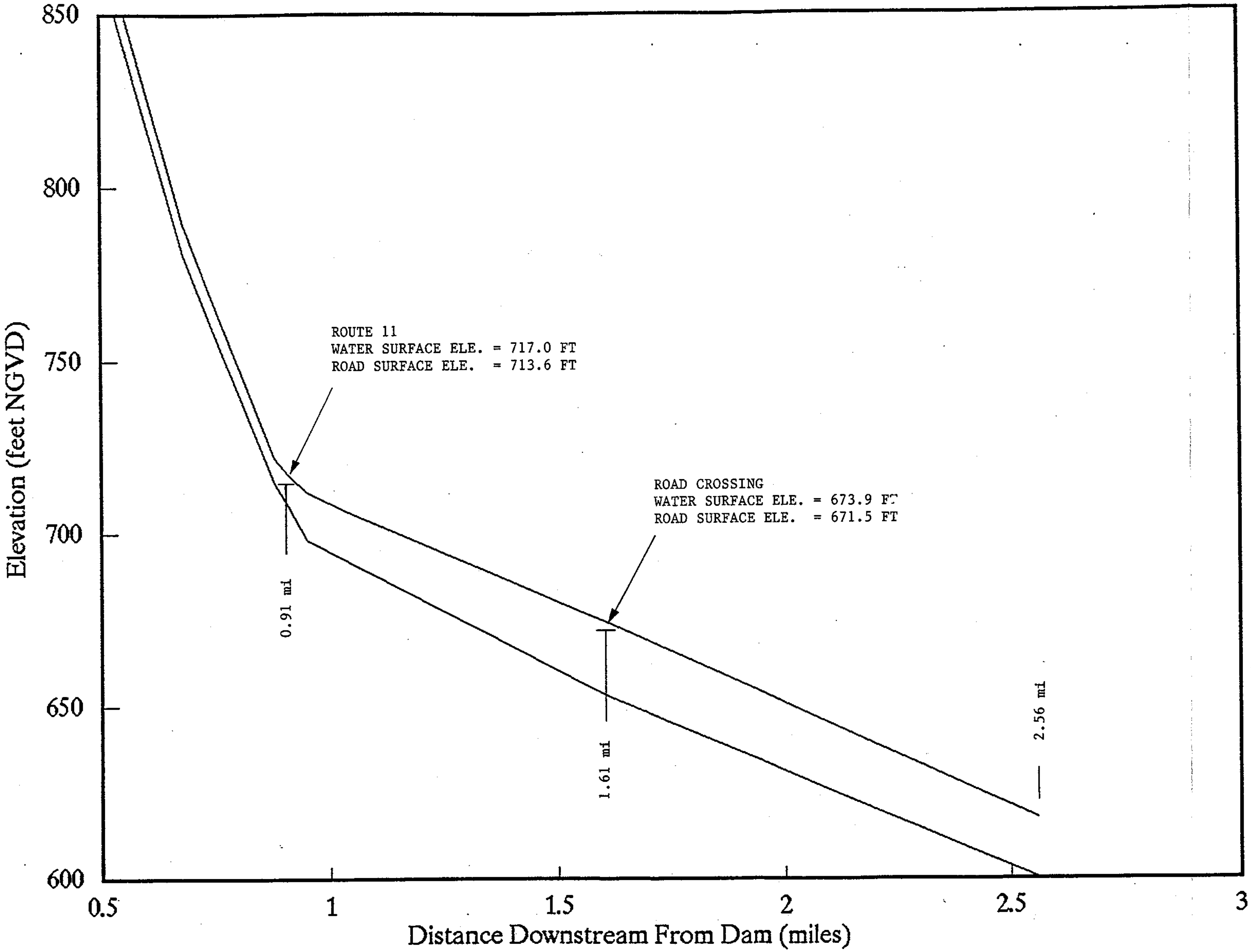

Figure 7. (continued)

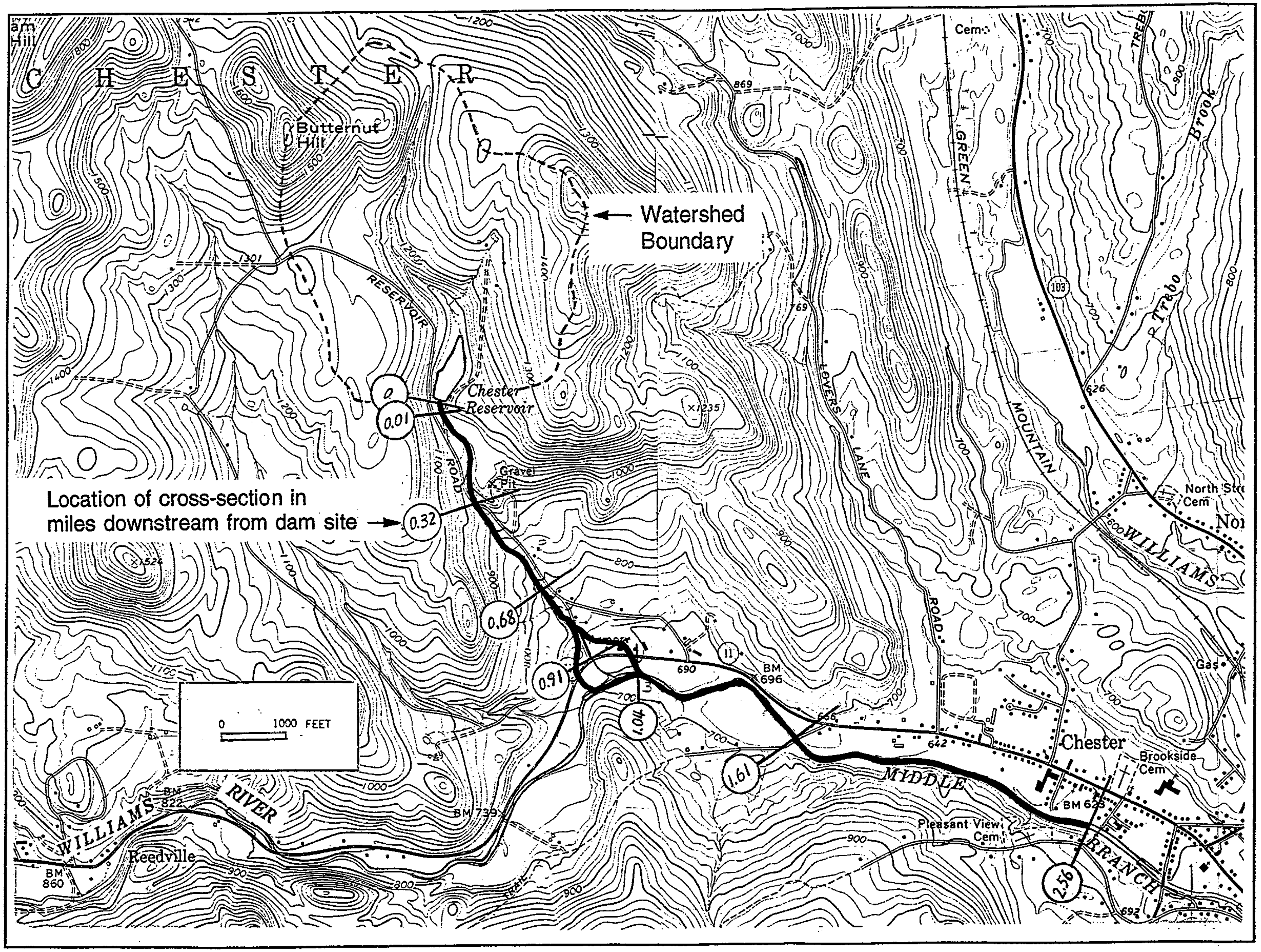

Figure 8. Inundation Map for Sunny-Day Failure Scenario

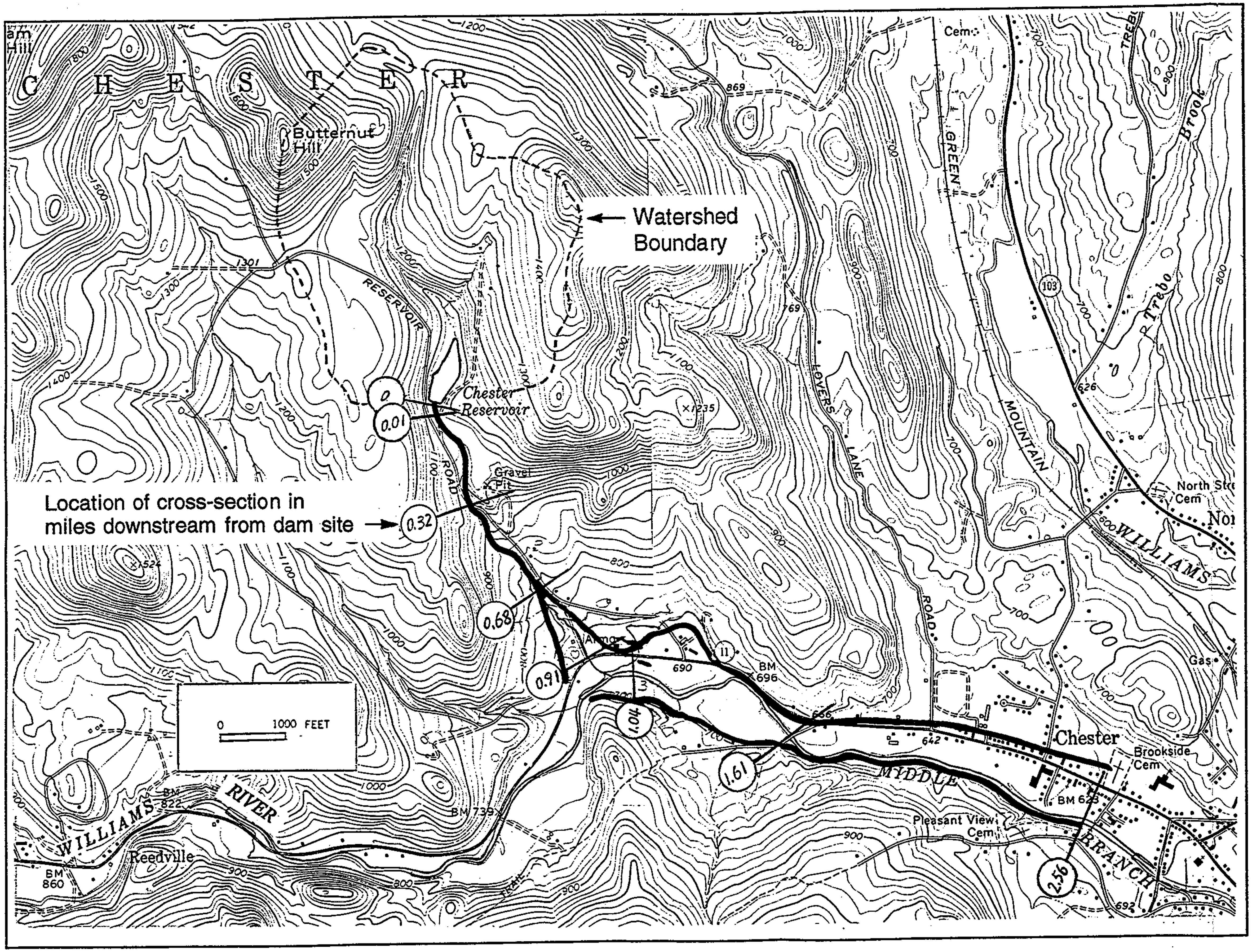

Figure 9. Inundation Map for Storm-Day Failure Scenario

B. EMERGENCY ACTION PLAN

1. INTRODUCTION

The Emergency Action Plan (EAP) is a suggested procedural outline (Ref. 1) indicating appropriate steps to be taken in the event of an impending failure of the Chester Dam. Also, this EAP lists phone numbers of certain local and state officials to contact in case of an emergency.

2. ITEMS IN THE EAP

The following are major items which should be addressed by the owner of the dam:

- Monitoring
- Evaluation
- Preventive Action
- Warning

3. MONITORING

a. Purpose

Having a person monitor the dam in the event of an impending dam failure is the first step in implementing the EAP. During periods of heavy precipitation, flooding or any unusual hydrologic events that might cause structural damage to the dam, the owner should have qualified personnel monitor the dam. The owner should assume responsibility for having the monitor at the dam within a reasonable time and for providing an adequate communication system between monitor and local officials.

b. Designated Owner Contact

Name:	Mr. Angelo Incerpi Director of Operations Department of Fish & Wildlife	
Phone:	Home:	(802) 684-3809
	Work:	(802) 241-3700

c. Training

The owner should provide proper training such that the monitor will have sufficient ability to recognize the condition of the dam and be able to identify and evaluate specific problem areas. This training should be extensive enough to allow the monitor to describe condition to local officials.

d. Communication System

The owner should provide primary and secondary communications systems between the dam monitor and local officials.

Primary System:	Normal telephone communication. The monitor should have access to the nearest available telephone and should have on his person the telephone numbers of all appropriate local officials.
Secondary System:	Shortwave radio: If the phone system is out of service, the monitor should have access to a shortwave radio that can be monitored by local officials with scanners.

As an alternative to this system, if any local officials live within a short distance of the dam, the monitor could drive to one of their residence if the roads are passable.

4. EVALUATION

a. Purpose

In conjunction with the ability to assess the condition of the dam, the monitor should have the ability to determine and evaluate the nature of any existing problem. This evaluation is a crucial step, because failure to accurately and promptly identify problem may adversely affect the EAP warning system.

b. Checklist items

Following is a check list of items that the monitor should use for assistance in preparing a safety assessment of the dam.

(1) Water Surface Level

Elevation:

a) Normal
b) High (if So, how high, with respect to the top of dam?)

(2) Principal Spillway

Condition upon arrival:

a) Clear
b) Blocked (if so, to what extent?)

(3) Emergency Spillway

Condition upon arrival:

a) Clear
b) Blocked (if so, to what extent)

(4) Top of Dam

a) Cover
b) Erosion

(5) Downstream Face

a) Cover
b) Erosion
c) Evidence of piping

5. PREVENTIVE ACTION

The monitor should ensure that the principal and emergency spillways are kept clear of debris during normal conditions. In the event of flood conditions, the monitor should also take reasonable steps to ensure that the spillways do not become blocked with debris so that they can carry their full capacity. The monitor's safety should not be jeopardized.

6. WARNING

a. Purpose

If the monitor feels that possible failure of the Chester Dam is imminent, he should immediately notify the designated parties by utilizing previously established communication systems. The monitor should notify the following officials and the downstream residents. Others can be contacted if determined necessary by the monitor.

b. Notification Chart (As of March 1994)

(1) Mrs. Sandra Walker
Town Clerk
Home: (802) 875-2637
Work: (802) 875-2173

(2) Mr. Gilbert Carey
Chairman - Board of Selectmen
Home: (802) 875-2807
Work: (802) 875-2173

(3) Mr. Prentice F. Hammond
Town Manager
Home: (802) 875-2381
Work: (802) 875-2173

(4) Mr. Harry Goodell
Fire Chief
Town of Chester
Home: (802) 873-2373
Work: (802) 875-3200

(5) Mr. Richard Crowson
Police Chief
Town of Chester
Home: (802) 875-2594
Work: (802) 875-2233

(6) Vermont Emergency Management Agency
24 Hour Duty Officer
1-800-422-8606
(802) 244-8721

(7) Town of Chester (Town Manager: Prentice F. Hammond)
Owner of Dam
(802) 875-2173

Official at the Vermont Emergency Management Office can be reached (24) hours a day. During normal business hours, the receptionist at the office will locate the current duty officer. During all other hours the phone connects to the Vermont State Police Department in Guilford, Vermont, which will locate the duty officer. In the event that the phone system has failed, any Vermont State Police barracks or cruiser can reach the duty officer through its radio system. Any available shortwave radio or CB radio could be utilized to contact the nearest police barracks.

c. Downstream Residents

(To be filled out and periodically updated by dam owner)

Name	Phone Number

C. REFERENCES

1. Recommended Guidelines for Safety Inspection of Dams, U.S. Dept. of Army, Office of the Chief of Engineers, Washington, D.C., Sept. 1979.

2. U.S. Weather Bureau Rainfall Frequency Atlas of the United States, May 1961, Technical Paper 40.

3. Hydrometeorological Report 51, the U.S. Weather Bureau Probable Maximum Precipitation Estimates, June 1978.

4. Hydrometoerological Report 52, the U.S. Weather Bureau Application of Probable Maximum Precipitation Estimates, August 1982.

5. HEC-1, Flood Hydrograph Package, User's Manual, September 1990.

6. DAMBRK, the NWS Dam-Break Flood Forecasting Model, Users Manual, November 1981.

7. Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains. USGS Water-Supply Paper 2339, 1989.